PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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A Typical Landslide in West Virginia

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BUREAU OF PUBLIC ROADS

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LANDSLIDES AND THEIR RELATION TO HIGHWAYS

A REPORT OF OBSERVATIONS MADE IN WEST VIRGINIA, OHIO, AND SOUTHWESTERN PENNSYLVANIA

Reported by GEORGE E. LADD, Associate Economic Geologist, United States Bureau of Public Roads

PART 2

THE earlier report dealing with this subject[1] discussed the geological characteristics of the regions in West Virginia and Ohio where landslides occur and the causes of their occurrence. This article will deal more particularly with the engineering problems involved and the various methods of control which have been tried, some of which have been successful and others complete failures.

Since the preparation of the first article some study of slides has been made in southwestern Pennsylvania, which is geologically closely related to the area studied in Ohio and West Virginia. The area in Pennsylvania indicated in Figure 1 is a part of the great geosyncline extending through West Virginia. Sandstones are less abundant than in the area covered in the first studies and shale predominates. The hills are more gently rounded and the predominance of shale has led to prominence of clay in weathered blankets on the hillsides. Water is freely absorbed by this material and slides occur under moderate loads and on very gentle slopes.

AVALANCHE TYPE SLIDE IN LOGAN COUNTY, W. VA.

No study has been made in a related area in northeastern Kentucky. The conclusions presented here pertain specifically only to the shaded area of Figure 1, although they may be occasionally applicable elsewhere.

Before describing several typical landslides it may be well to summarize briefly the conditions which have led to landslides as pointed out in the previous article. These conditions are listed as follows:

Where a cut has been made in tilted strata on the down-dipping side in such material as sandstone, limestone, and solid shale beds.

Where a vertical or nearly vertical face has been left in a cut in solid shale, jointed shale, or jointed shale and sandstone, and material is precipitated as a result of frost action or slippage on interbedded seams of water-softened clay.

Accumulation on hillsides and in gulleys on the sides of high hills of detritus consisting of clay and silt from decomposed or weathered shale, highly rounded sand, and sandstone fragments and bowlders, which make a more or less fluid mass when sufficiently wet. Detritus of this character is also found in parts of river flood plains as a result of old slides and causes trouble.

Beds of plastic or fire clay, especially near the surface.

Plastic clay coatings formed on slopes beneath detritus by the wetting and softening of comparatively pure shales.

Artificial fills made of materials of the general character described above.

It has been pointed out that water plays a very important part in causing slides both by lubricating and increasing the weight of material. The effect of direct rainfall is often combined with seepage to precipitate slides, and under certain conditions direct rainfall on an area is a sufficient cause. Materials subject to sliding may be moistened sufficiently to start them in motion either directly or by combinations of the following: Rainfall on an extensive surface of detritus; rainfall penetrating plastic clay beds near the surface of fills; rainfall accumulated in long, steep gulleys filled with detritus; underground water flowing through joint planes in sandstone and along the top of impervious shale beds; underground water passing through shattered shale beds or through shales that are very sandy and somewhat pervious; water from streams during floods, wetting the base of fills, or an underlying detritus blanket; and rain water accumulated and stored in pockets of old slides.

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[1] Landslides and their relation to highways, Public Roads vol. 8, No. 2, April, 1927.

Water may also reach material in unstable equilibrium from artificial sources such as accumulations in coal mines and in quarries, especially where rock seams have been opened by heavy blasting; it may be let into sidehill fills by joints in culverts which have opened as a result of settlement; it may be admitted through broken culvert connections or from side ditches during prolonged rains.

A few typical slides will serve to illustrate the manner in which these several conditions contribute to the movement of material and the success in preventing the movement that may be expected of various corrective measures.

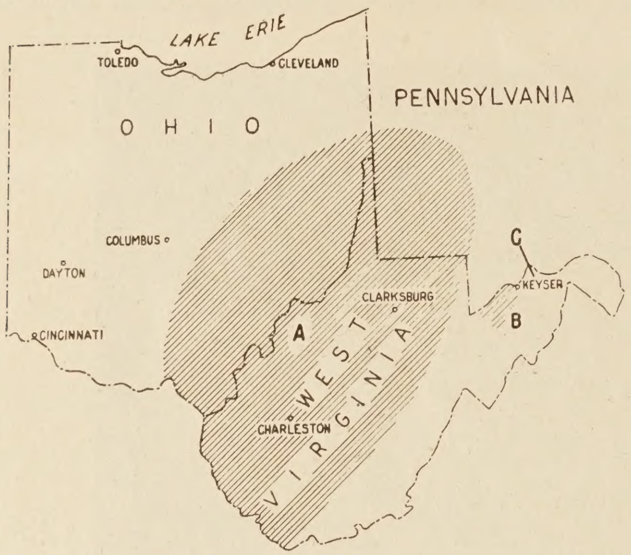


Fig. 1.—Sketch Showing Area Covered in Study of Landslides

AVALANCHE TYPE OF SLIDE REMOVED BY STEAM SHOVEL

In the vicinity of Lyburn, Logan County, W. Va., in the spring of 1927, a landslide occurred on a hillside where a cut had been made partly in shaly sandstone and partly in overlying detritus. Where the slide occurred the material was all detritus. This slide was 200 feet wide at the bottom, measured along the road and extended up the hill for a distance of over 800 feet. At the upper end it divided and detritus moved from two forks.

The highway is on a narrow bench about 60 feet above a branch of the Chesapeake & Ohio Railroad which, in turn, is about 20 feet above the Guyandot River. The slide buried the highway to a depth of 25 feet or more, and, crossing the road, buried the Chesapeake & Ohio tracks below. Steam shovels at each end of the slide worked for 30 days opening the highway for traffic. Trucks had to haul the detritus 2 miles at one end and 7 miles at the other. Ten thousand cubic yards were removed from the highway. The railroad was blocked for about four days until track could be relaid upon the bench widened by the slide.

The slide was of the avalanche type, rare in this area, and except for a small subsequent movement was over in less than two hours. The detrital material contains an extremely small amount of clay, a great deal of rounded sand, and an abundance of sandstone fragments and bowlders.

It caused considerable indirect damage, especially to the railroad which normally handles a large tonnage of coal over this branch. During this tie-up 15,000 people were supplied with food by carrying supplies over the slide, with truck delivery at both ends.

The fact that the cut was in detritus for the entire width of the slide, considered with topographic evidence, indicates original development of a gully by ancient slides and the subsequent filling by numerous lateral slides. This one occurred following a heavy rain which saturated the porous detritus, lubricated the small amount of clay and rounded sand particles, added the weight of a large volume of water, and so started the movement down the mountain side which at this point has a slope of about 45°.

Except in suddenness of occurrence, this slide typifies those which come upon the road and leave a large yardage to be removed by steam shovel. The direct costs to State and railroad were over $30,000. Further slides do not appear to be impending in the immediate future, but the mass of material above the road indicates that slides will recur at some future time.

SLIDE TEMPORARILY CONTROLLED BY LOG CRIBBING

Near Barnabas, Logan County, W. Va., a slide moved down onto a road at intervals for four years. As in the Lyburn slide, there was an exceptionally thick accumulation of detritus in a ravine formed by an old slide. Such an accumulation can very easily occur in the sandy, rocky materials of the local detritals. A large water supply reaches the area of the slide because of topographic conditions and the volume retained is great as a result of unusual thickness of detritus and its porous nature. This slide necessitated removal of 32,000 cubic yards of material with steam shovels.

Following the last clean-up, cribbing was installed to check the slide. White-oak logs 12 to 14 inches in diameter at the butt and 12 feet long were placed in a manner similar to that illustrated on page 162. The tie logs extending back into the slide were at intervals of 4 to 5 feet. Slope adjustment of the detritus, which began shortly after the setting of the cribbing, buried everything but the face of the cribbing, but the movement was finally halted. Previous expenditures at this point by the county have amounted to $17,300. The cribbing described above cost $2,200 for logs and $2,500 for labor. The slide is now apparently controlled; but the control is only temporary as the cribbing timbers will rot and it is probable that the detrital mass will again move upon the highway.

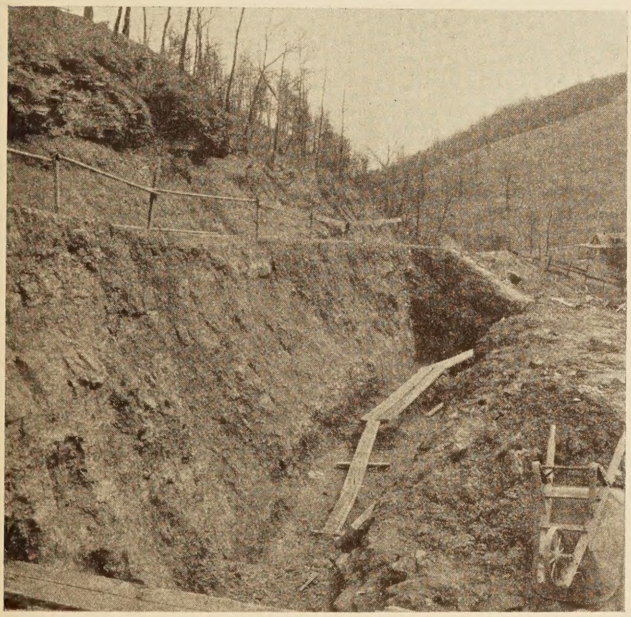
RETAINING WALL HOLDS SIDEHILL FILL

In Green County, Pa., between Washington and Waynesburg, the highway traverses a hillside for a considerable distance. The surface material is decomposed shale, which freely absorbs water, as evidenced by crawfish holes in fields on both sides of the road. In many places side fills built of this unstable material go out and undermine the pavement. At one or two points drainage methods which will be described later have been employed for control of such subsidences, but in most instances where trouble has been encountered reinforced concrete retaining walls have been built on the lower side of the road. These walls have held the road in place. One typical wall is 150 feet in length, 13 feet high, and rests upon a 9-foot footing which is set from 3 to 5 feet in solid shale. Such a wall costs about $8,500, or about $57 per running foot.

RETAINING WALL PLACED WHERE CHEAPER DESIGN WOULD HAVE BEEN ADEQUATE

Federal-aid project No. 143–A, West Virginia, winds among high hills between West Union and Parkersburg. During grading operations 2 miles west of West Union it was found that the fill shoulder would not stand. The fill descends to a meadow, which is badly drained and very wet. The slide was not attributed to capillary water from the meadow, as it was thought that both the fill and the meadow were supplied with seepage from a somewhat previous shale or from a coal seam within it. A standard type concrete retaining wall was built 4 feet from the edge of the pavement.

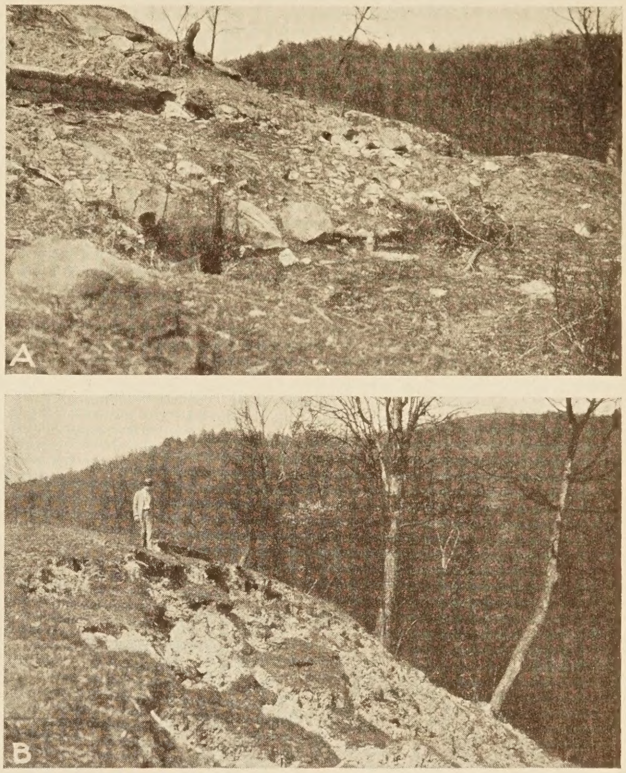
The illustration below shows the conditions at this location. More or less solid shale extends in places nearly to the level of the road, but it is not as compact as most shales in this district. It is quick to disintegrate under weathering influences, as shown by the retreat of its face beneath the ledge of sandstone above the road. The retaining wall has no surcharge, and considering the amount of material to be held by it the cost seems unduly high. Well casing could have been deeply set in the shale extending beyond the foundation for the retaining wall (see illustration) and filled with reinforced concrete. If spaced closely, they would form a thin and light type of wall, which would have cost much less than a retaining wall. Such a wall could have been back-filled with sandstone taken from the ledge above the road, and, with proper weep holes, having no serious pressure to withstand, could easily have held the road shoulder. The writer was among those responsible for the construction of a retaining wall and shares fully in the responsibility for it, but wider observations of similar cases are convincing that there is a cheaper and quicker way.



Excavation for Retaining Wall to Hold Road Shoulder, near West Union, W. Va.

SATURATED FILL OVERTURNS RETAINING WALL

On West Virginia Federal-aid project No. 19, at a point known as White Rock on the West Fork River, a retaining wall was built on the river side to hold a side fill in place. The highway is located about 50 feet above the river and the wall was founded on solid rock which outcropped along the river bank near the low-water mark. The gravity-type wall was keyed into the rock in the usual manner and was approximately 60 feet in length, 15 feet in height, and 6 feet thick at the base. A fill consisting of some shale and rock, but mostly of weathered detritus, was placed back of the wall and extended from its top to the highway on a slope of 45°. Drainage channels were left in the wall.



A Slide in Putnam County, W. Va., which Overturned a Retaining Wall

After completion of this wall there was a period of high water in the river which lasted for several days, and the water rose to approximately the top of the wall. It then fell rapidly, and within a few days was back to normal. Shortly afterwards the wall toppled over into the river. There was no breaking of the concrete. The wall remained intact, lying on its side in the water.

The explanation is simple. During the flood stage water flowed in through the weep holes and around the ends of the wall, thoroughly saturating the fill back of it. This caused no damage as long as the river was up, but when it fell to normal stage it removed the compensating pressure from the face of the wall; and the saturated material back of it, which exerted a more or less liquid pressure against the concrete, plus the effect of the surcharge of the fill above, caused the failure.

Another wall failure occurred in Putnam County, W. Va., where a cement-rubble retaining wall, 200 feet long, set in shale about 75 feet beyond, and well below the road shoulder, went out during a rainy spell. The cause in this case was apparently an excessive surcharge of the saturated fill above. A number of natural slides occurred at the same time in the vicinity, one of them about 100 feet from the toe of the wall.

SLIDING FILL PROVES DIFFICULT TO CONTROL

About 1912 Cabell County, W. Va., took over a section of abandoned railroad right of way from the Chesapeake & Ohio Railroad between Barboursville and the Putnam County line east of Huntington. On this grade the county built a 16-foot grouted brick pavement on a concrete base. The State road commission of West Virginia took it over for maintenance on January 1, 1922.

The road crosses a deep ravine on a fill about 450 feet long at Lee's Creek, 2½ miles east of Milton. During the winter of 1924–25 a section of the fill near the west end started to move from under the pavement on the north side. In the spring the movement increased and it was necessary to support the pavement by tamping rock and other material under it. A section about 40 feet long and half the width of the pavement finally failed.

In September, 1925, widening of the fill on that side was begun. In order to compact the new fill as much as possible, it was put in with wheel scrapers, starting at the toe of the old fill. The new fill was carried up about 10 feet wide and was kept level, the teams traveling the whole length of the fill in going to and from the borrow pit.

This work was completed the latter part of September. During the winter of 1925–26 the new fill slid off at the same place where the old fill had settled, carrying still more of the old fill with it. About this time it was noticed that the ground at the toe of the fill was rising in front of the slide. This continued, as the movement of the fill progressed, until a section averaging 20 feet wide and extending about 100 feet along the toe of the fill had been lifted from 3 to 6 feet above its original elevation.

In the spring of 1926 churn-drill soundings were made parallel with and at the toe of the fill. Rock was found at elevations varying from 4 to 12 feet below the ground surface, the latter elevations being near the center of the fill.

It was decided to set well casing filled with concrete in this stratum. A well drill was used to sink 8-inch holes through the detritus and 8 feet into the rock, which seemed to be a fairly hard sandstone. Well casings, 6⅝ inches inside diameter, were set in these holes and filled with concrete. Where the rock was more than 8 feet below the surface, four ½-inch reinforcing rods were set in each casing. One hundred and twenty of such casings were set on 3-foot centers. After these casings were completed, the slope of the fill was trimmed and ditched and no pockets were left to catch water.

During the winter of 1926–27 the fill continued to settle, although to a somewhat less degree than during the two preceding winters. Early in the spring of 1927, 12 of the casings where the rock was deepest showed signs of bending over. By the summer of 1927 they had failed completely. These were the casings which had been reinforced. The fill continued to sink from under the pavement in a section 40 feet long. The

FILL SLIDE AT LEE'S CREEK, W. VA.

break which had previously extended only to the center of the pavement now covered nearly the whole width. No sign of slipping appeared on the opposite side of the fill.

In April, 1927, a core drill was used to sink a line of holes parallel to the curb and about 2 feet from it across the entire north side of the fill. Nothing but fill material and detritus was encountered to a depth of from 38 to 52 feet below the pavement except a few scattered bowlders. The greatest depth to rock was in the bed of Lee's Creek, which crosses the fill about one-third of its length from the east end.

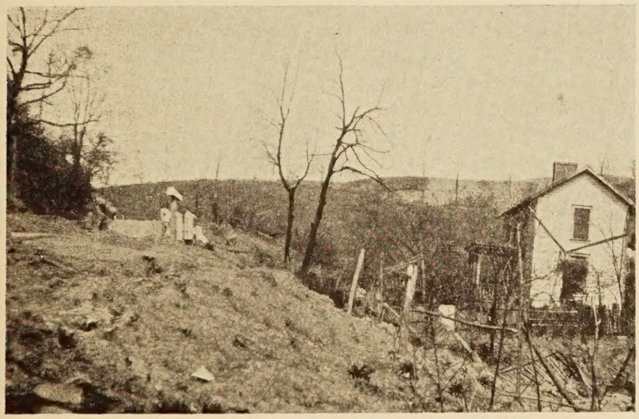
In August, 1927, one hundred and twenty-two 25-foot oak piles were driven parallel to the pavement and about one-third of the distance up the slope from the casing. These were spaced 3 feet apart. In October the fill was brought to the proper cross section by addition of new material. This material was distributed by hand so as to have the slope as uniform as possible.

At present the slide movement seems to have stopped. There is still a little settlement in the pavement, but this is to be expected as normal settling against the piling.

It is probable that this fill will continue to give trouble and that slumps and subsidences will recur, in spite of any physical-force methods of control, until the water problem is solved. A sluggish stream flows through a swampy tract near the foot of the fill. Old detritus and stream-deposited muck underlie one side, at least, of the fill. These are water-saturated in part from the stream, and in part from a water-bearing seam somewhere beneath the fill at its hillside contact and it is believed that a permanent remedy lies in drainage rather than attempts to hold back the fill.

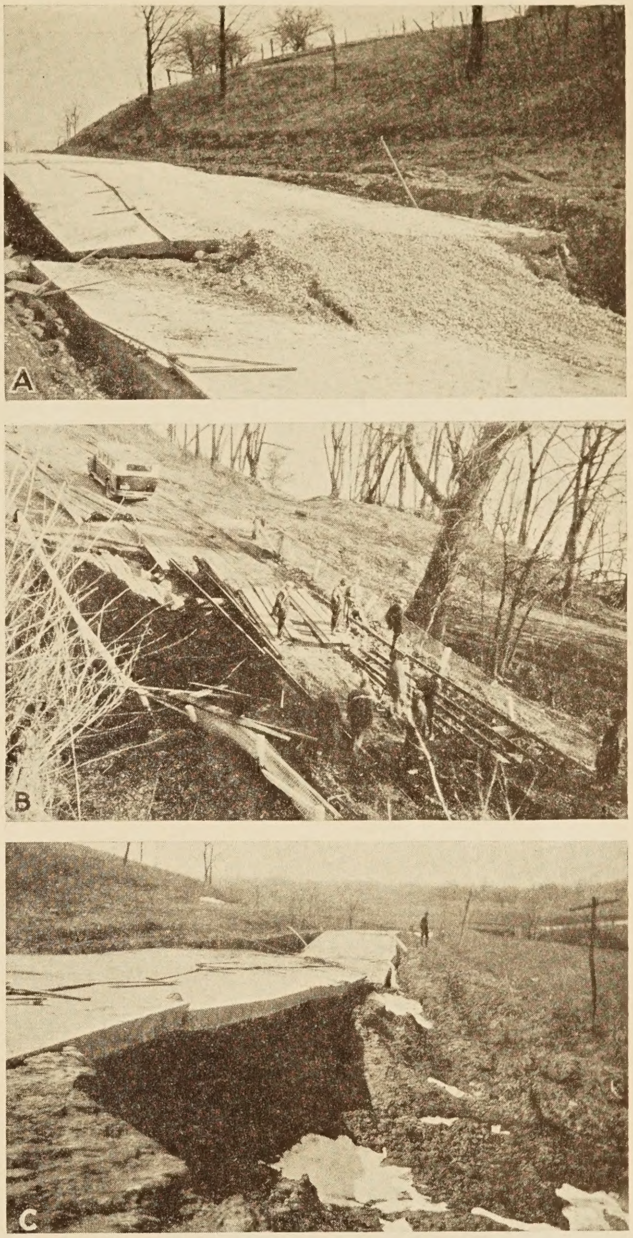
FILL SLIDES OF VARIOUS TYPES DESCRIBED

In southwestern Pennsylvania a concrete highway was constructed on a hillside. Part of this road crossed old slide material and at one point the grade line required a fill of 2 or 3 feet on the uphill side and considerably more on the downhill side. The outer shoulder was built of shale and decomposed shale or siliceous clay. This fill extended down to a garden plot in front of a two-story frame residence erected on old slide material. A short distance beyond this residence the land drops sharply for a short distance to a small stream.



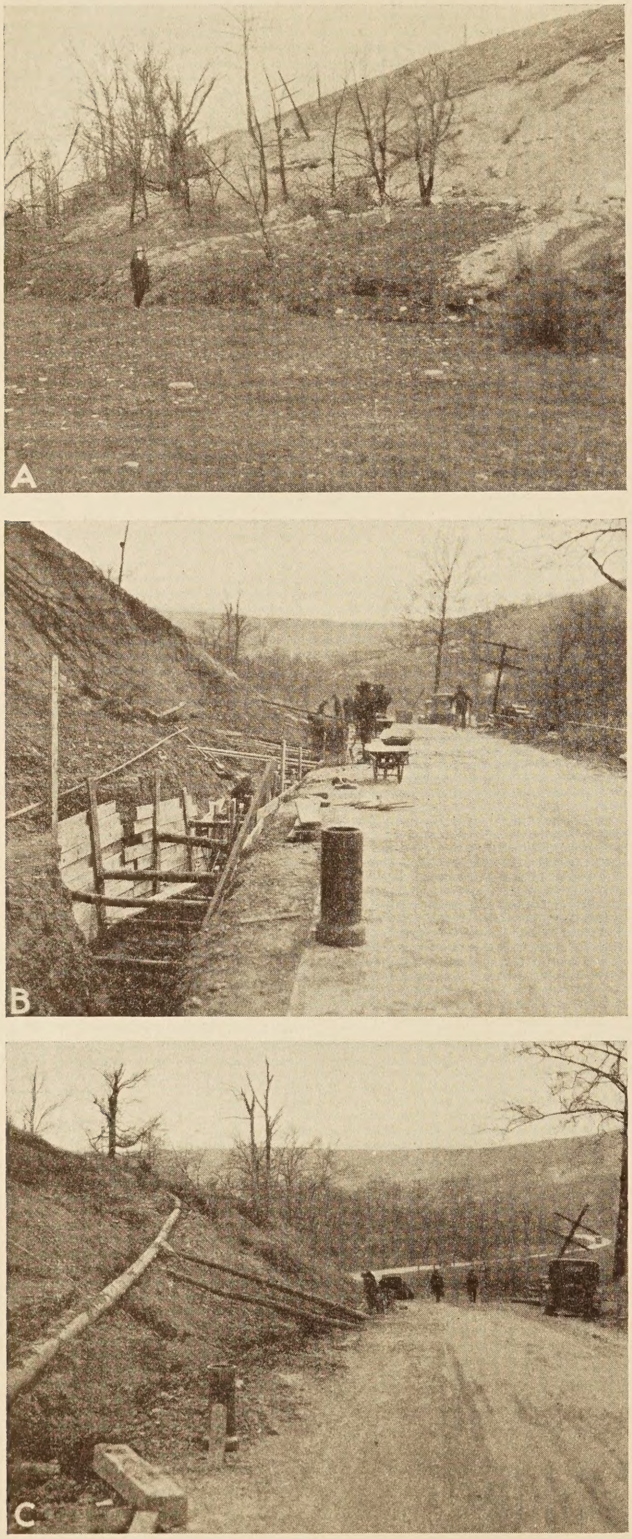
An Old Slide Started in Motion by Weight of New Highway Fill of only Moderate Height

Not long after construction was completed a section of pavement dropped about 6 feet vertically and the pavement was dislocated laterally for a distance of 200 feet or more. A domelike bulge arose in the garden and the whole mass of the old slide moved sufficiently to break the foundation of the house, leaving the basement without walls on two sides, threatening the house and making it unsafe for occupancy. The construction of the highway added additional weight on the old slide but did not change conditions otherwise. The original mass must have been barely stable and a moderate load was sufficient to start it in motion.



Side Fill Slides and Resulting Damage to Highways

The old slide was evidently fed by seepage through a noncompact shale near the road level. There are hundreds of cases similar to this in character. They are so frequent as to make imperative a study of local conditions before construction is begun. Such a study should include observation of wet spots following rainy spells, but after a general superficial drying of the surface. Where seepage is indicated, ample drainage must be established on the inside of the road.

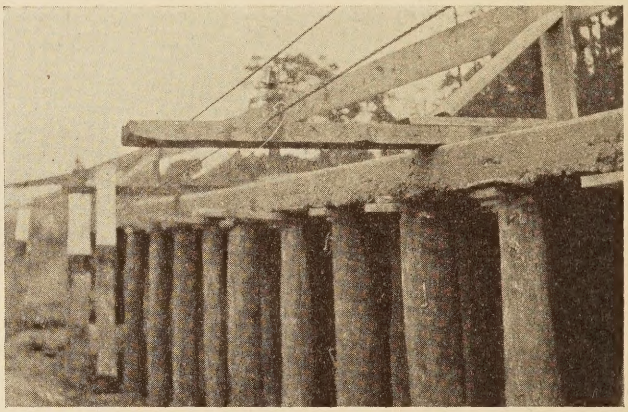


Views of Slide and Corrective Measures on Bell Hill near Morgantown, W. Va.

In southwestern Pennsylvania a fill was constructed which contained 9,000 cubic yards. It was 60 feet deep at the lower toe and 25 feet at the upper toe. The material was decomposed, disintegrated clay taken from an adjacent 18-foot cut through laminated clay. The lower layer of the cut was quite plastic. Grading was begun in the winter of 1925 and was completed about the first of the following May.

Ten days after completion of the work the whole lower side of the fill went out, carrying about half of the roadway, which settled about 20 feet. It was evident that the fill material would flow readily when wet, but there was no evidence of seepage or sufficient rainfall to cause a slide.

A probable explanation of this failure is that the fill was built of clay, much of which was very plastic in the winter time. The lower part of the fill was constructed with wet clay, much of which was frozen. A considerable load was placed on this material and when the winter and rains came the load was increased by moisture. The frozen wet clay thawed, the bottom flowed out, and the fill collapsed.



A Concrete Road Supported on Posts after a Sidehill Slide

A retaining wall was built at the toe of the slide and the original 42-inch culvert pipe, 172 feet long, was extended to a length of 200 feet. About 1,000 cubic yards of rock placed on the side of the fill as it was built up, presumably with the idea of holding it in place, merely added to the weight on the underlying mucky ground and caused it to overflow the retaining wall and rise 4 feet above its top.

A few miles west of Clarksburg, W. Va., a sidehill fill slid out from under a considerable portion of a concrete pavement. A row of concrete posts was installed to support the pavement, the backfilling was tamped in, and the road shoulder was refilled. The concrete posts were set in old detritus rather than in solid material; water continued to penetrate the fill; and it again went out, taking with it material from beneath the supporting concrete posts.

It is probable that deep but relatively inexpensive drainage installed inside of the road would have safeguarded the fill and pavement. The entire section of road was relocated, however, so as to place it on a solid shale.

DRAINAGE PROVES EFFECTIVE IN STOPPING SLIDE

One of the most interesting and instructive cases of slide and fill subsidence occurred on a highway on Bell Hill about 1 mile east of Morgantown, W. Va. An old county road descended the hill to a small valley where a bridge crossed a stream. The old road had required a small cut in shale and some underlying detritus on the hillside and the material had been thrown over the side and constituted a moderate fill. A number of years passed without record of noteworthy trouble.

When the road was improved by the State as a Federal-aid project it was thought best to locate it further into the hillside on more solid material. The new road was placed 10 feet back in the hillside, the bridge approach was raised 6 feet, and a new bridge was constructed. The hillside cut furnished material for the wedge-shaped fill downhill to the bridge, and a large surplus which was dumped on the hillside fill making a very wide shoulder. The load on this fill was increased by building a high bank on the outer shoulder to serve as a guard for traffic. All of this was dumped on old slide material, which, however, was berm, and did not involve the area to be paved.

The road was graded in the summer of 1925, and in the fall or winter of 1925–26, a period of wet weather set in which was followed by a break extending diagonally across the road at a point about halfway down the hill. The location of this break is indicated in Figure 2, by the letter A. The road settled almost vertically, below the break. The bottom land below the slide showed unmistakable signs of many old slides indicated by rolls or slight hillocks. After the road settled it was observed that the land near the bottom began to upheave and there were indications of considerable pressure apparently transmitted from the weight of the fill to the plastic subsoil below.

A slip also occurred in the sidehill fill at the point marked B in Figure 2. After the slip occurred a stream of water could be seen emerging from the solid material approximately 8 feet below the surface of the road. This water had entered the loose fill on the hillside and so saturated the earth that it became semiliquid, and settled down into the bottom on an angle of repose not exceeding 20°. At the top of this slip the ground rose almost vertically to the surface of the road, 8 feet above the point where the water emerged.

Discovery of this stream of water led, quite naturally, to control by drainage. A ditch was dug along the upper side of the road which extended from the top of the grade two-thirds of the distance toward the small stream. Flowing water was encountered at a depth of approximately 8 feet. The ditch was carried approximately 10 feet deep and into solid rock, and an 8-inch tile drain with open joints was laid on a gradient of approximately 3 per cent. The ditch around the tile was filled with broken stone and over this several feet of cinders were placed. The remainder of the ditch was filled in with earth. This drain intercepted the water as it flowed from the rock and discharged it into the open ditch near the small stream. The drain effectually stopped the flow of water into the slip below the road. Even in dry periods the discharge of water from the drain was sufficient to fill a 2-inch pipe. The fill along the lower side of the road (at point B) was then replaced to form a shoulder, and this fill has shown no signs of settlement since that time.

The slip, marked A, Figure 2, was filled in with cinders and appeared to be stable. In the late fall of 1926, however, a large water main supplying the city of Morgantown, which was located on the upper side of the road, broke as a result of a slide above the road and for a number of hours discharged a large volume of water into the slip and over the road at A. This water thoroughly saturated this slip and much settlement took place in the week following the break. As the road settled it was brought back to grade with cinders, and it was used as a cinder road during the winter of 1926–27. In the spring of 1927 it was observed that very little settlement or slipping had taken place during the winter months. Careful observations were taken over a period of four months in the spring and practically no settlement was observed. A concrete surface was then laid. Shortly afterward a settlement of about 2 inches took place along the lower side of the road, but no further settlement has occurred up to the present time.

It is believed that the breaking of the water main thoroughly saturated the slip and caused the maximum settlement for such material. There has been subsequent drying out and no further settlement has taken place excepting some slight readjustments.

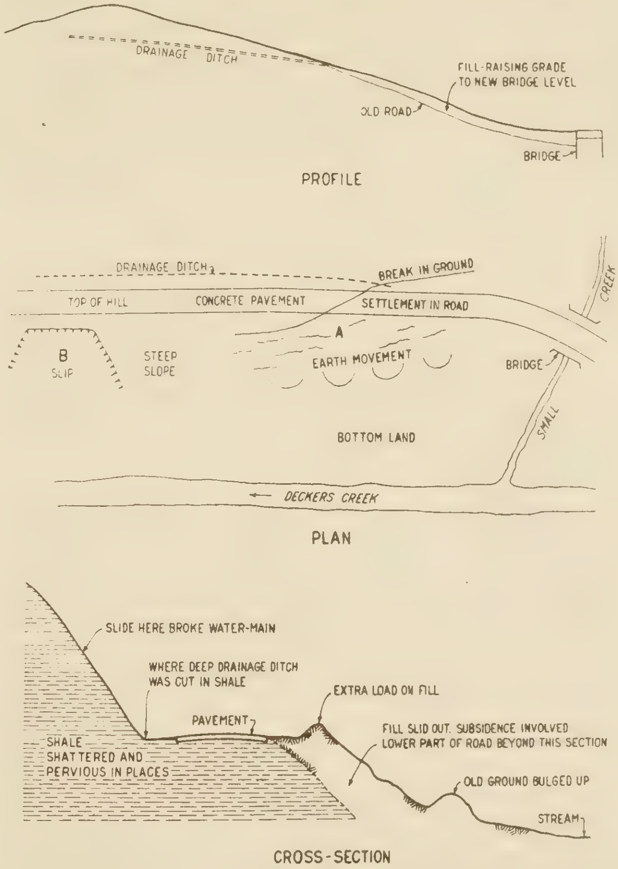


Fig. 2.—Showing Conditions Surrounding Slides on Bell Hill near Morgantown, W. Va.

FALL OF DÉBRIS A SERIOUS PROBLEM

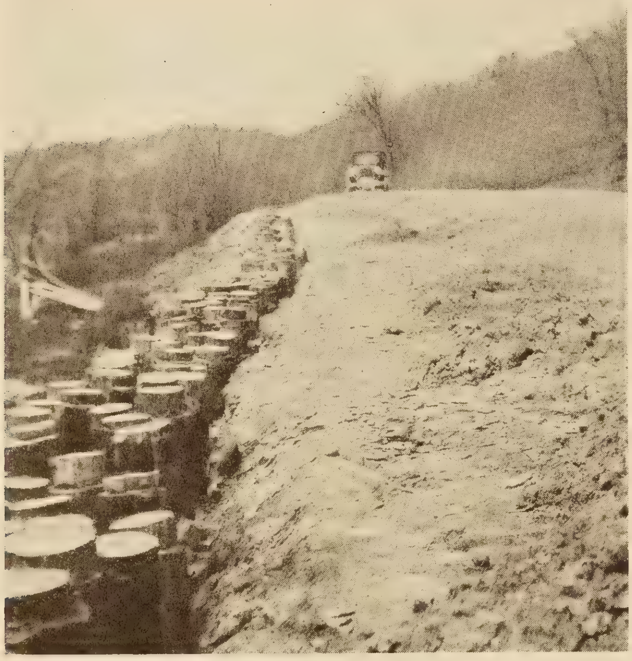
Slide material precipitated in ditches and on roads is generally removed by steam shovels and trucks, but sometimes hand shoveling and trucks are used. Dribble constitutes the greater part of such material and its occurrence is very widespread. Eight steam shovels have recently been purchased for work of this character in eastern Ohio.

Often the cuts made in massive shale leave a vertical face rising to a considerable height above the road. If the shale is jointed, it is inevitable that frost action in time will precipitate large masses on the road below. Sometimes a broken condition results from heavy blasting during excavation. Similar dangerous conditions are also found in sandstone.

Vertical cuts may be left in shale if it is compact, without joints, and has not been deeply shattered by blasting. Otherwise, where possible, a suitable slope should be provided during construction. It is cheaper and safer to remove the surplus material then than to have to remove it from the pavement, and run the risk of physical damage, interference with traffic, and possible loss of life, at a later time.

The fall of much of this small material is not economically preventable, but attention should be given to bowlders and outcrops which endanger highway traffic. Projecting and unsupported sandstone and detached bowlders resting in an unstable position should either be pried loose or blasted and deposited below the highway.

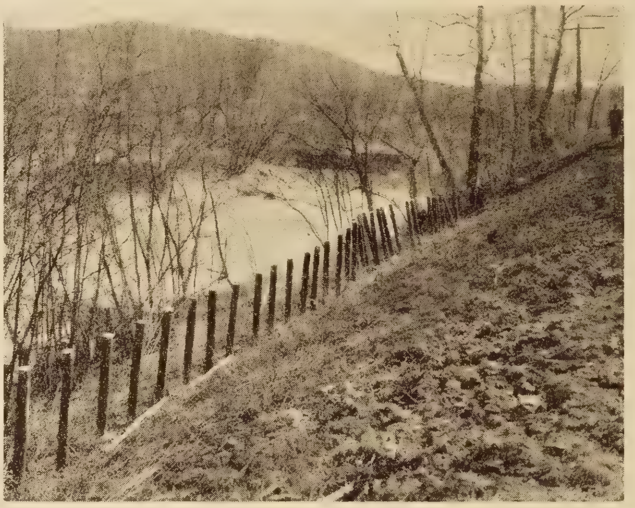
Cuts should not be made in strata dipping steeply toward the road unless absolutely necessary. This is especially true if the rock has seams of clay or shale, even though only an inch or so in thickness. Where such a cut is made, rock beds above should be anchored in place in advance of blasting by a system of reinforced concrete piles set in drill holes.



Closely Spaced Piling to Hold a Road in Place

USE OF WOOD PILES HAS DISADVANTAGES

The use of wood piling is very extensive. It may be found in use in attempts—temporarily more or less successful—to hold fills in place and to hold masses that have moved or threatened to move on a road. There are cases where a pavement rests in part upon old slide material and in part upon fill. Often the pavement is threatened with undermining and dislocation, and with being covered by material from above. In such cases piling has been placed at the foot of the fill and at the upper side of the road. As many as five rows may be seen at some points. Occasionally, for a single purpose, as many as three rows are placed. In some cases they are staggered, on 3 to 5 foot centers, with rows 4 feet apart. Frequently the piling is driven as close together as possible. In the illustration below a row of piling may be seen so massed as to constitute a retaining wall. Oak piling used for this purpose varies from 20 to 30 feet in length. A good and general practice is to drive it to refusal into an underlying bed of solid shale. Occasionally piles are used wholly in fill material to stiffen the mass but this practice has not been very successful.



Casing Filled with Reinforced Concrete Used to Retain a Fill. Oak Piling Nearer the River has Failed and the Concrete Piling is Beginning to Fail. Drainage of a Swampy Area on the Other Side of the Road Appears to be the Remedy

H. J. Spelman, division engineer of the West Virginia State Road Commission has stated[2] that the cost of such piling in his division when done by contract varied from $0.75 to $1 per lineal foot of timber. Many jobs have been reported as having cost approximately $0.90 per lineal foot of timber.

The use of wood piling is believed to have several disadvantages. It is never known in advance whether a single row will prevent the movement of a sliding mass. The sight of old piles either completely overthrown or pointing out from hillsides like cannon is not uncommon. After one row has been put in, if there is evidence of further movement, another row may be added and so on up to a maximum of five. Five rows may cost from $28 to $33 per lineal foot of road. The life of untreated wood piling, and this is almost universally used, is relatively brief. The United States Forest Service estimates it at approximately eight years.

CASINGS FILLED WITH CONCRETE OFTEN USED

The use of piling formed by filling well casings with concrete has increased in popularity and is becoming extensive. The cost in this district is somewhat less than it would be elsewhere as second-hand oil and gas-well casings can usually be purchased. The concrete filling is sometimes put in without reinforcing. Steel rods are the common reinforcement but in a few cases steel rails have been used. Mr. Spelman gives the cost of such casing with reinforcing, at $1.11 per lineal foot of casing; but, on many jobs it has cost $1.20 or

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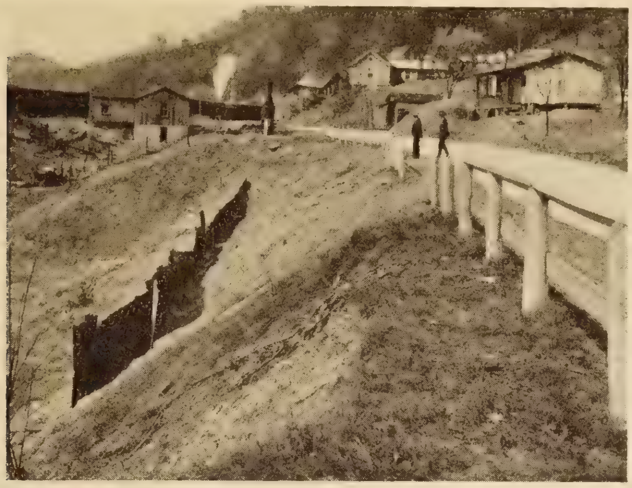
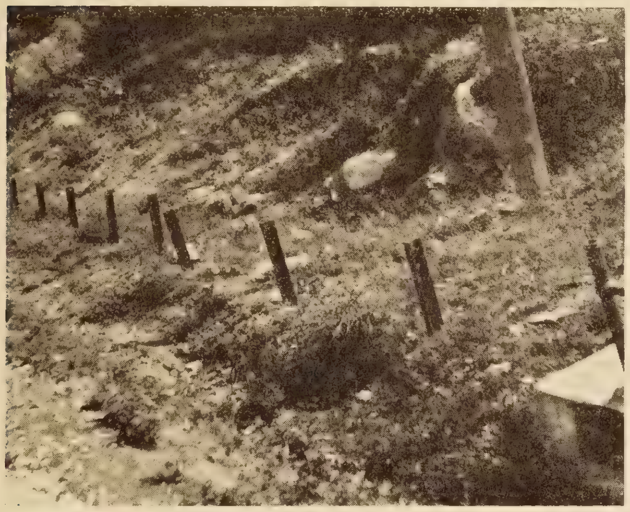
[2] Engineering News-Record, Mar. 17, 1927, p. 438.



Sheet Piling Used to Hold a Fill



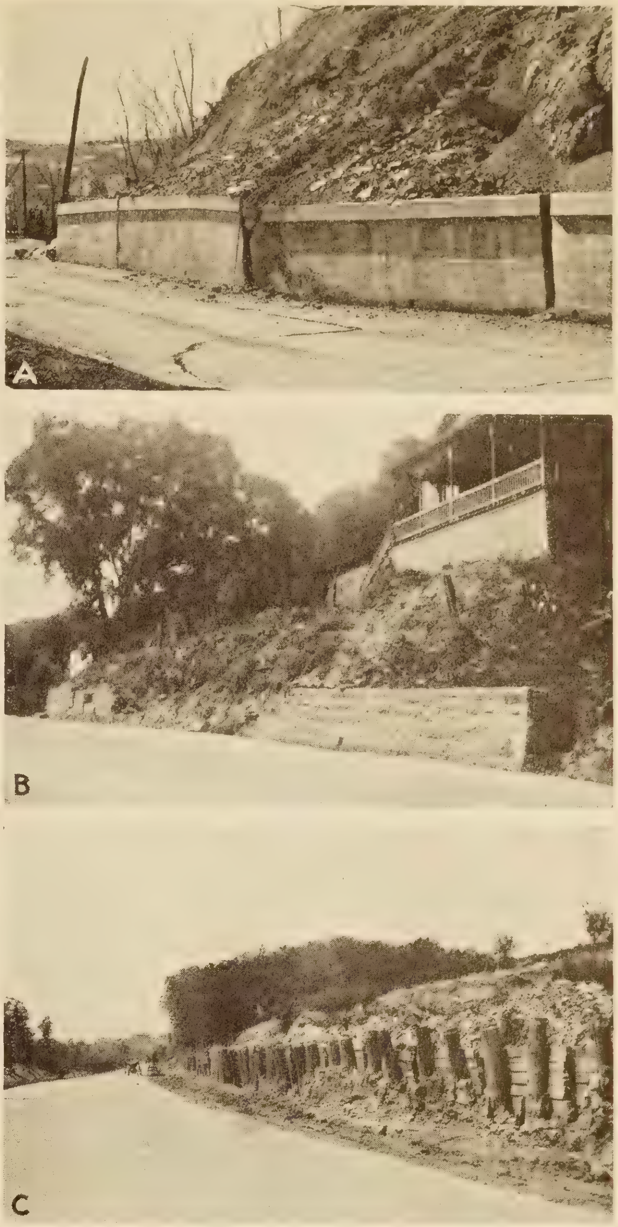
Steel Rails Used to Hold a Slide

more per lineal foot. No overhead is included in these figures.

This type of piling is often set from 5 to 8 feet deep in so-called solid material which is usually shale or sandstone. Both this type of piling and hardwood piling are often backed with planking, especially near the top.

The use of casing filled with concrete to hold any considerable mass of sliding material is always experimental. The piles may hold, and they may not. Sometimes rows of piling have gone out, probably because they were placed too near the edge of buried shale or rock which split under pressure.

In West Virginia, at a point known as the Godbey slide, the paved road which was buried is slightly above a small river. A clean-up was made, and to avoid future trouble detritus was removed for a considerable distance beyond the inside shoulder of the road. This cut exposed a vertical face of solid shale 12 or 15 feet in height, evidently made by the river earlier in its history. This condition shows remarkably well how casing might be set near the edge of such a face and result in failure. It is important that the outline of solid material be ascertained before such control methods are undertaken.



Types of Retaining Wall Used to Protect Road

There are numerous failures where the cause is unknown. Casings may have been set in shale which softened, allowing them to tip over. They may have been bent or broken. It would be worth while to pull a few such piles to get more definite information.

SHEET PILING AND STEEL RAILS USED

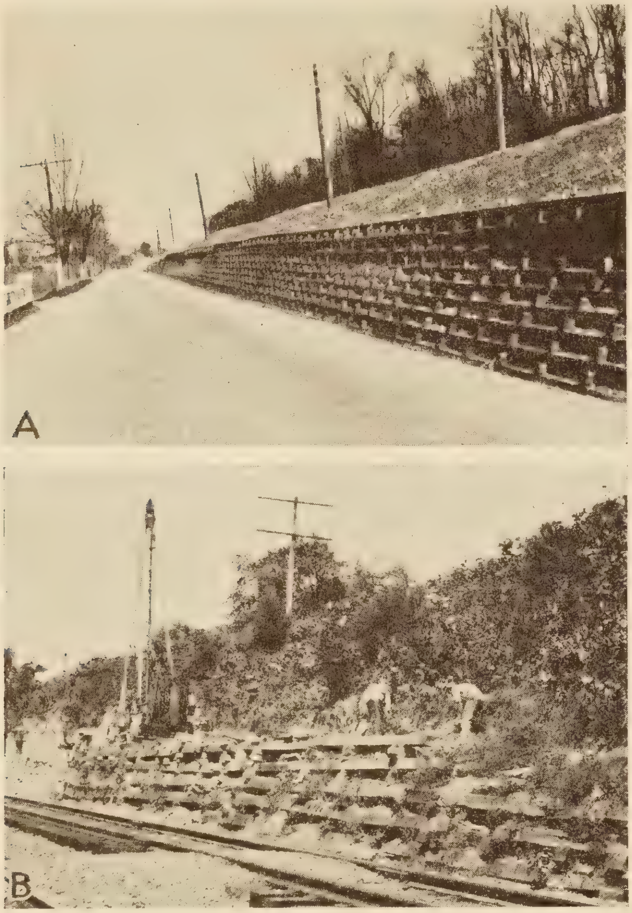
Only one case of the use of sheet piling has been observed. On this job the sheet piling appeared to have stabilized the mass above it which was already nearly stable, but it has not prevented high water in the stream below from penetrating the fill, which was the intent of the experimental undertaking. The cost per lineal foot of such piling is intermediate between that of oak and reinforced concrete.

Rows of steel rails have occasionally been used by railroads for the control of slides, and in one or two instances by highway engineers, but not very satisfactorily. A case of such use is illustrated above.

RETAINING WALLS EXTENSIVELY USED

Retaining walls, although relatively expensive, are extensively used, mostly on the lower side of the pavement, but not infrequently above the road and sometimes to protect residential property and cemeteries.

Many cases have been observed where the cost ranged from $35 to $60 per lineal foot and where less expensive protection such as casing filled with concrete would serve as well. Usually proper drainage on the inside of the road and beneath it will be cheaper and more satisfactory in results than a retaining wall.



Timber Cribbing for Slide Control

A strong objection to the relatively expensive retaining wall lies in the uncertainty as to the magnitude of the forces that will act on the wall.

ROADS OFTEN RELOCATED TO AVOID SLIDES

As new highway projects develop, engineers in this district are concerning themselves more and more with the problem of relocation at obvious points of danger. The question presented is that of balancing the present cost of more or less extensive grading against the probability of sooner or later losing part of a paved road. Occasionally extensive changes in plans seem imperative.

CRIBBING SUCCESSFULLY USED

The use of cribbing at the foot of a slide mass after a steam shovel clean-up is increasing. Cribbing has usually been built of untreated rough oak logs. Lately squared creosoted logs have come into use, and, very recently in Ohio, concrete units have been tried. The initial cost of the two latter types of cribbing is high, but the results obtained are likely to be permanent.

The illustration of cribbing on this page shows a case where the timber has become so rotten as no longer to prevent detrital movement. The old cribbing is being removed and reinforced concrete piling placed in its stead. It is not safe to depend on cribbing to retain an unstable mass which extends beneath the road. In such a case if a brute-force method seems necessary, cribbing must be supplemented by piling. In typical cases observed, oak cribbing cost $10 and concrete cribbing $38 per lineal foot of road.

BLASTING SELDOM PRODUCES WORTH-WHILE RESULTS

Drilling through slide masses and near the toe of fills which are moving, or in which incipient movement has started, and dynamiting the underlying material is a time-honored method of attempted control. It is often recommended by engineers and in the area under discussion has been resorted to by some highway engineers. It has been used more extensively by railroad engineers in connection with side-fill slides. Some of them have adopted the practice of drilling holes 35 to 55 feet in depth, extending 18 to 25 feet into so-called solid material along the toes of fills. Holes are spaced 20 feet apart, and after springing sometimes twice with 30 to 60 sticks of 40 per cent dynamite loads varying from 300 to 500 pounds of 40 per cent dynamite are shot in them.

There does not appear to be any evidence to justify the use of this method except in special cases. When used it is stated that it is experimental or that it opens up underground drainage and allows water to escape. Again, it is claimed that it roughens the ground beneath the fill and prevents sliding. This theory assumes that a fill slides as a unit on an underlying surface, whereas, in nearly every case, there is movement throughout the mass or, at least, the lower part of it. The movement is a slump rather than a slide.

If drainage channels are opened by blasting, relief is likely to be only temporary, for they will quickly be choked with fill material. In one case investigated holes were drilled to a depth of 17 to 20 feet below the level of an immediately adjacent river and shot. It was stated that the advantage to be obtained was undersurface drainage.

In several instances shale base lying below water level has been heavily blasted. In such cases it is not possible to avoid the belief that the blasted material will rapidly disintegrate into muck. Such practices can only result in very temporary control and must ultimately lead to the destruction of the only basis for rational control methods.

Under special conditions, however, there may be some advantage in blasting underlying solid sandstone or limestone.

ROCK LOADS OF LITTLE VALUE IN PREVENTING SLIDE

Several cases have been observed where large bowlders were placed at random on the outside of side fills. Usually the bowlders so placed gradually work to the toe of the fill. In a number of cases masses of bowlders have been assembled originally at the toe of a fill. The accompanying illustrations show two cases where a rock load is used to supplement piling.

Bowlders distributed over a fill, apparently with the idea of holding the fill down by load, add nothing to the cohesive strength of the mass; they simply add further to the load, which is already too great. Bowlders massed along the toe of a fill probably do exert some retaining effect, but on the whole their use is of little value.

DRAINAGE BELIEVED TO BE SOLUTION OF PROBLEM

The most important conclusion resulting from the study of slides in this district is that, generally speaking, preventive measures should be substituted for retaining structures. Usually a certain load is unavoidable. The nature of the detrital material is such that it is unstable when wet, the tendency toward instability depending on the fineness of the material and the clay content. The normal load and nature of the material can not generally be changed, but it is usually possible to eliminate the third factor causing slides, namely, water. It is not necessary to eliminate all moisture, but the content must be kept below the critical point at which it makes the mass unstable.

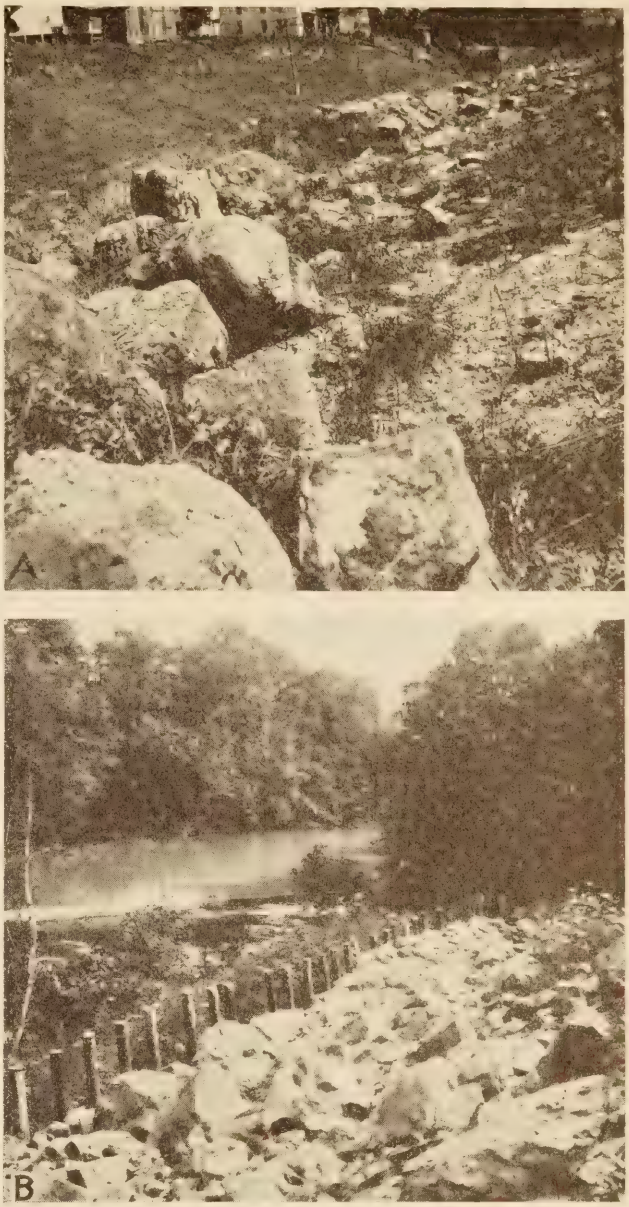
This may be done, according to the nature of the problem, by surface protection from penetration by rainfall, by surface drainage, or by underground drainage which reaches the source of seepage or flow. Side fills and through fills can be protected from penetration by rainfall where necessary. It is also possible to isolate them from underground water in most cases. Masses of overhanging detritus can generally be sufficiently drained to prevent movement.

Only sporadic attempts at drainage control have been made. A case at Morgantown, W. Va., has been described where drainage of underlying shale solved a serious problem. Small-size, open-joint tile has been occasionally used in soft spots in a road and French drains have been placed beneath inside ditches. Some surface drainage has been undertaken for the purpose of removing water from pockets of overhanging, old-slide detritus. Such work, however, has been neither systematic nor thorough. Lack of emphasis on prevention is common to most human experience and we spend money on landslides largely after they have occurred.

It is believed that this district needs more trenching machines, and fewer steam shovels and piles; that drainage will be found to be cheaper and more permanent than any control method now employed; and that it must be undertaken with knowledge of local geological conditions. Detrital areas which are traversed by roads must be studied from a geological standpoint. Water seepage must be traced to its source, and water volume determined following rainfalls of varying intensity. Test holes or other means of interior exploration will answer this purpose.

If detrital material has been undisturbed for a considerable time, fine clay may have been washed downwards and accumulated below as in the formation of subsoils. Therefore, where slide material has been at rest for some time, the greater part of the underground water will be found comparatively near the surface, that is within 3 to 5 feet of it. In one case observed after a heavy rainfall, where a cut had been made in a thick mass of detritus, water was escaping in almost a solid sheet, along a plane about 5 feet below the top of the cut.

Before fills are placed it is vitally important to observe whether the location is on ground which is wet not only during but for some time after rains. When wet spots are found, the source of the moisture must be located. Frequently it is in detritus on the hillside and after a fill is placed water enters from the side or end contact. The use of wet materials in building a fill, especially at or near its base, has been demonstrated to be a dangerous practice.



Masses of Rock Placed at Toe of Sliding Fill to Supplement Piling

A number of illustrations of sidehill failures are presented because they constitute the most serious phase of the slide problems in this district. Most of them can be prevented by drainage on the inside of the road, or, better, by drainage installed before the road is graded.

The annual damage resulting from the slides and subsidence is so enormous that systematic preventive experiments and study of relative costs and permanency of results is obviously justified.

It is believed that the solution of a very large proportion of the cases which arise, and this includes evidence of danger as well as slide movement, lies in the direction of drainage.

THE MODULUS OF ELASTICITY OF CORES FROM CONCRETE ROADS

RESULTS OF DETERMINATIONS MADE ON CORES DRILLED FROM MARYLAND HIGHWAYS

By **A. N. JOHNSON**, Dean of Engineering College, University of Maryland

THIS is the second report dealing with the general subject of strength characteristics of concrete based on investigations carried on cooperatively by the Engineering Experiment Station of the University of Maryland, the United States Bureau of Public Roads and the Maryland State Roads Commission. The first report[1] described the method of taking cores from concrete roads of various ages on the Maryland State system and analyzed the results of compression tests on such cores.

The general plan of the investigation included extensive determinations of the stress-strain relation for the cores in order to study their elastic behavior. One or two specimens were selected from each group of cores and set aside for such tests. The best possible end condition was secured by placing a thin plaster-of-Paris coating on the caps which had been placed upon the cores for crushing-strength tests, a flat surface being secured by the use of a piece of plate glass put on while the plaster was soft. Elastic measurements were made on these cores in the testing machine with the load applied by hand at a much slower rate than was possible when the crushing strengths only were being obtained using power loading.

ACCURACY OF DIFFERENT TYPES OF EXTENSOMETERS INVESTIGATED

The type of extensometer first used was what may be called a 5-point extensometer as it clamped the specimens at five points.[2] After considerable experience with this instrument it was evident that it was unsatisfactory, the stress-strain curves becoming more and more irregular with increased use of the instrument. It was then determined to make a comparison between this extensometer and a simple form of mirror extensometer.[3]

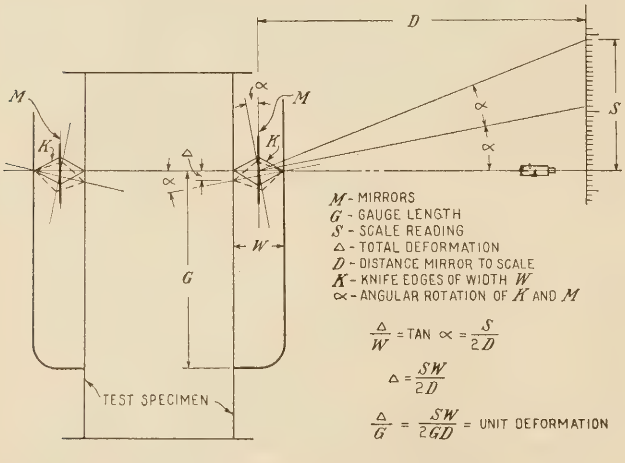

D
M
M
K
K
α
α
α
α
Δ
S
W
G
M - MIRRORS
G - GAUGE LENGTH
S - SCALE READING
Δ - TOTAL DEFORMATION
D - DISTANCE MIRROR TO SCALE
K - KNIFE EDGES OF WIDTH W
α - ANGULAR ROTATION OF K AND M
$\frac{\Delta}{W} = \tan \alpha = \frac{S}{2D}$
$\Delta = \frac{SW}{2D}$
$\frac{\Delta}{G} = \frac{SW}{2GD}$ = UNIT DEFORMATION
TEST SPECIMEN

FIG. 1.—DIAGRAM ILLUSTRATING MIRROR EXTENSOMETER

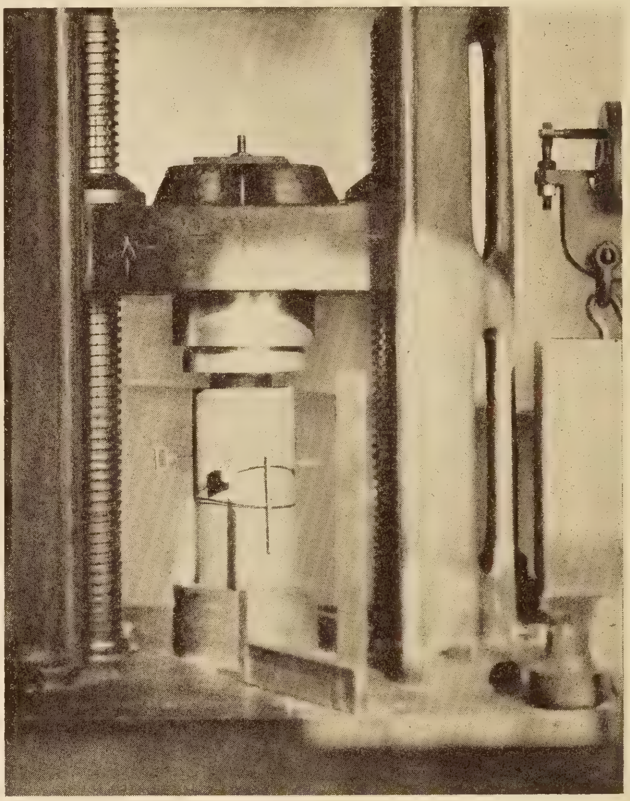


FIG. 2.—CONCRETE CYLINDER IN TESTING MACHINE WITH MIRROR EXTENSOMETER ATTACHED

The principle of the action of the mirror extensometer is shown in the diagrammatic sketch of Figure 1, while Figure 2 is a photograph of the set-up. In Figure 1, the geometric relations are clearly shown. It is evident that what may be called the base line of the measurements made with the mirror extensometer is the width W of the knife edge shown in the sketch. If this is accurately measured, there is no difficulty in securing the other measurements with as much accuracy as is necessary. The knife edges actually used were made as closely as possible to 0.2 inch. They were calibrated by measuring with a microscope with the proper micrometer attachment. One of the knife edges was found to be 0.2015 inch and the other 0.2012 inch. The distance D from the mirror to its scale (one for each mirror) was adjusted so that 1 inch on the scale corresponded to a deformation of 0.001 inch. The scale was divided into tenths of an inch, so that measurements quite accurate to one hundred-thousandth of an inch over a 4-inch gauge length, or 2½ millionths inch per inch, were possible.

Since the support for the scales was independent of the weighing table of the testing machine which supports the specimen, it is evident that there would be indicated on the scales both the deformation of the specimen, and the movement of the specimen with

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[1] Strength Characteristics of Concrete as Indicated by Core Tests, Public Roads, vol. 9, No. 7, September, 1928.

[2] A description of this extensometer may be found in the Proceedings of the A. S. T. M., vol. 19, pt. 2, p. 510 (1919).

[3] A description of this extensometer may be found in Martens' Handbook of Testing Materials.

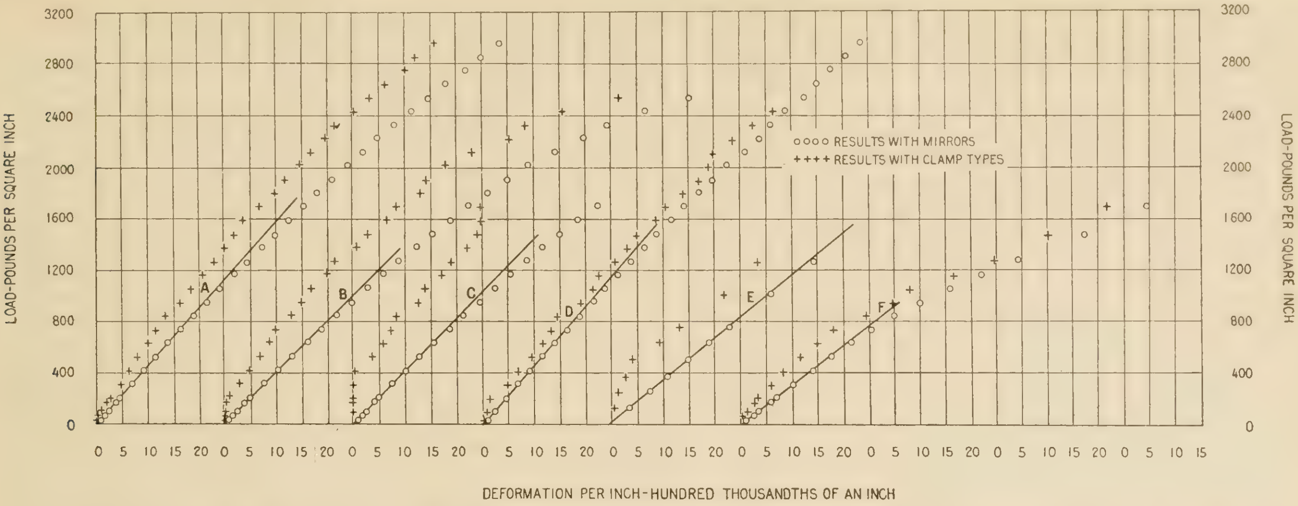

FIG. 3.—COMPARISONS OF LOAD-DEFORMATION CURVES OBTAINED WITH CLAMP EXTENSOMETERS AND MIRROR EXTENSOMETERS

reference to the scale. It was possible to so arrange the apparatus that this error was automatically eliminated. The mirrors were mounted as shown in Figure 2 and both faced in the same direction so that as the specimen shortens under compression, the right-hand mirror throws the beam of light upwards on the scale (fig. 1), whereas, the left-hand mirror throws the beam of light downwards (the scale for the left-hand mirror is not shown in the diagram). Therefore, a given movement of the specimen with reference to the scale will have the effect of increasing the reading of the right-hand mirror, but the reading of the left-hand mirror will be decreased by the same amount. The average of these two readings, obtained from the mirrors revolving in opposing directions, exactly cancels any effect produced by any vertical movement of the specimen with reference to the fixed scales. With this method one mirror is not sufficient, and two must be used, giving readings in opposite directions. Also at least two mirrors must be used to give the average strain in the specimen.

MIRROR EXTENSOMETER FOUND SATISFACTORY

The results with the mirror extensometer were so divergent from those obtained with the 5-point extensometer that further comparisons were made between other types of clamp extensometer and the mirror type. There was a marked difference in the characteristics of the curves obtained with the two types of instruments as is shown graphically in Figure 3. The data shown upon this figure were obtained by simulataneous observations with a clamp type of extensometer and a mirror type, the circular dots showing the results with the mirror type, while the crosses are the results with the clamp type.

It is to be noted that the clamp type of extensometer invariably gives a curve with increased slope near the origin, and which flattens out toward the latter part of the curve. The explanation is that extensometers which are clamped to the specimen almost invariably have a certain amount of lost motion between the point of contact and the end of the dial needle, and until the deformation of the specimen is sufficiently great to take up such lost motion the curve obtained is not a correct indication of the deformation of the specimen, and the stress-strain curve that results is not an accurate interpretation of the behavior of the specimen under the applied loads.[4] This is particularly true for the lower pressures where the dial readings of the clamp extensometer give little indication, as may be seen in Figure 3, particularly in curves B, C, and E. This effect obviously is not dependent upon the character of the concrete, but is merely a mechanical defect in the measuring apparatus. Measurements that had been obtained with the clamp-type extensometer were therefore abandoned and only those obtained with the mirror type are here reported.

One marked characteristic that the mirror-extensometer readings give to the stress-strain curve for concrete in compression is the persistency of a straight line relationship in the earlier part of the curve for the lower pressures and the gradual departure from a straight line forming a very gradual curve for the higher pressures. The extent of the straight line relationship or proportionality may be approximately determined and is taken to be the elastic limit of the concrete.

With the use of the mirror extensometer erratic results, which are characteristic of the clamp type, were not obtained. In fact, so uniform were the curves as plotted from the readings of the mirror extensometer that if a point did not fall closely on the general trend of the curve there would usually be found some arithmetical error in reducing the observations and the correct value would place the point on the curve.

TEST RESULTS INDICATE BEHAVIOR AS AN ELASTIC MATERIAL

This report discusses the results of elastic measurements on 112 cores. The average crushing strength was 3,960 pounds per square inch, and the average value for the modulus of elasticity was 3,562,000 pounds per square inch. The lowest value of the modulus recorded was 1,200,000 pounds per square inch, and the highest 5,840,000. The individual values are given in Table 1.

This was discussed in the Proceedings of the A. S. T. M., 1924, pt. 2, p. 1026.

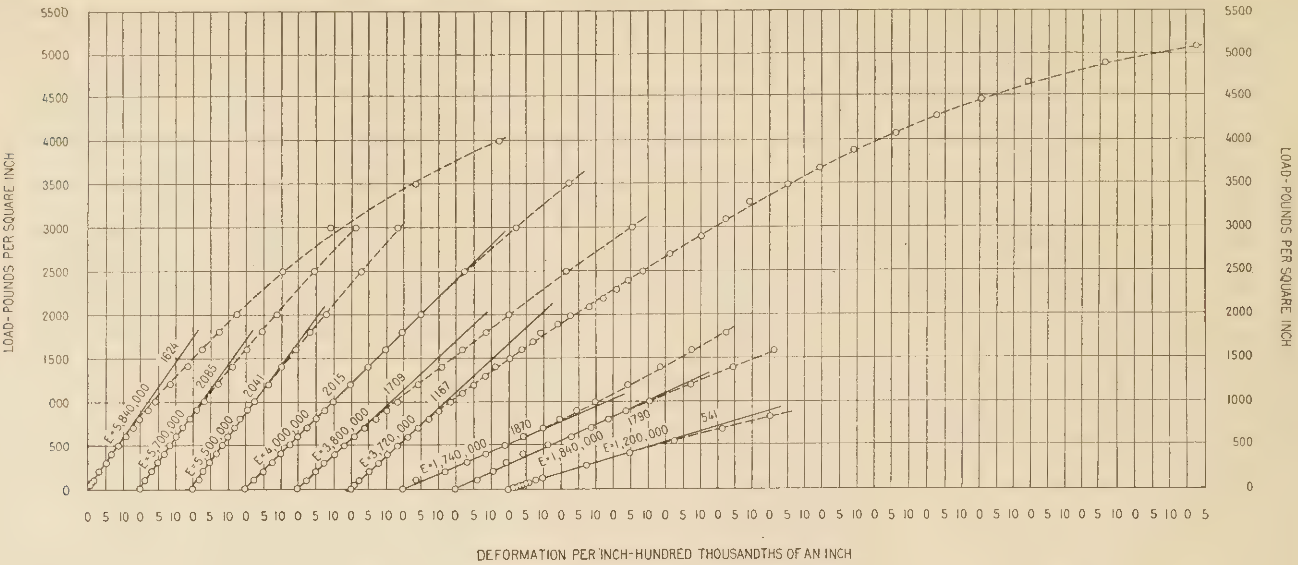

Fig. 4.—Load-Deformation Curves for Concrete Cores Tested in Compression

Table 1.—*Crushing strength and modulus of elasticity of concrete road cores*

Core No.	Age	Coarse aggregate	Crushing strength	Modulus of elasticity	Core No.	Age	Coarse aggregate	Crushing strength	Modulus of elasticity
	Years		*Pounds per sq. inch*	*Pounds per sq. inch*		*Years*		*Pounds per sq. inch*	*Pounds per sq. inch*
239	6	Mixed rock [1]	3, 000	2, 200, 000	1624	6	Trap and gravel	5, 100	5, 840, 000
240	6	do	4, 400	3, 180, 000	1637	6	Slag	3, 100	1, 900, 000
523	5	Limestone	4, 900	3, 540, 000	1678	10	Quartz gravel	2, 160	4, 040, 000
541	4	Granitic rock	2, 330	1, 200, 000	1680	10	do	5, 210	4, 860, 000
689	7	do	3, 200	3, 200, 000	1681	10	do	2, 360	4, 380, 000
690	7	do	3, 800	2, 100, 000	1682	10	do	2, 630	4, 480, 000
691	7	do	3, 800	4, 200, 000	1683	10	do	1, 700	3, 380, 000
693	7	do	3, 800	2, 500, 000	1684	10	do	4, 320	4, 020, 000
694	7	do	4, 800	2, 700, 000	1685	10	do	5, 110	4, 240, 000
696	7	do	3, 600	3, 500, 000	1686	2	do	3, 010	3, 150, 000
698	7	do	3, 800	3, 200, 000	1689	1	do	3, 000	3, 680, 000
699	7	do	2, 900	3, 240, 000	1690	1	do	2, 860	3, 400, 000
700	7	do	3, 000	2, 600, 000	1691	1	do	3, 000	3, 100, 000
701	1	Limestone	3, 000	3, 280, 000	1692	1	do	3, 850	2, 680, 000
703	1	do	3, 000	2, 400, 000	1693	1	do	3, 500	3, 860, 000
704	1	do	4, 900	4, 640, 000	1694	1	do	2, 300	4, 140, 000
706	1	do	4, 000	5, 300, 000	1695	10	do	3, 110	4, 070, 000
712	1	do	3, 200	2, 720, 000	1696	10	do	3, 280	3, 440, 000
716	1	do	2, 300	2, 480, 000	1697	10	do	3, 650	3, 700, 000
1076	½	do	4, 700	3, 680, 000	1698	1	Trap	2, 700	4, 240, 000
1087	2	do	4, 300	3, 960, 000	1699	1	do	3, 870	3, 970, 000
1097	2	do	3, 000	2, 920, 000	1700	1	do	2, 360	4, 100, 000
1108	3	do	5, 000	3, 820, 000	1701	1	Quartz gravel	5, 050	4, 240, 000
1120	1	do	6, 100	5, 200, 000	1702	1	do	3, 175	3, 120, 000
1145	1	do	5, 100	5, 200, 000	1703	1	do	2, 180	2, 480, 000
1155	½	Quartz gravel	4, 400	3, 680, 000	1705	½	do	3, 600	4, 560, 000
1167	4	do	5, 300	3, 720, 000	1706	½	do	2, 960	3, 610, 000
1180	6	do	3, 400	2, 480, 000	1707	1	do	3, 280	3, 960, 000
1192	8	do	2, 700	2, 040, 000	1708	1	do	4, 290	3, 700, 000
1208	8	do	2, 800	2, 300, 000	1709	1	do	4, 750	3, 800, 000
1222	1	Limestone	4, 000	4, 140, 000	1722	1	Limestone	2, 200	5, 100, 000
1246	1	do	4. 800	4, 460, 000	1723	1	do	4, 100	5, 640, 000
1259	7	do	4, 000	4, 260, 000	1724	1	do	5, 000	5, 420, 000
1271	7	do	4, 700	4, 900, 000	1730	1	Quartz gravel	5, 000	3, 300, 000
1286	6	do	2, 500	2, 740, 000	1764	1	Mixed rock	4, 610	4, 360, 000
1298	9	Quartz gravel	4, 400	2, 720, 000	1790	2	Quartz gravel	4, 320	1, 840, 000
1319	6	do	5, 800	2, 900, 000	1823	1	Limestone	4, 330	3, 220, 000
1330	2	Limestone	5, 900	4, 600, 000	1847	2	Quartz gravel	4, 070	3, 380, 000
1343	8	do	4, 300	3, 360, 000	1870	2	do	3, 160	1, 740, 000
1369	7	Granite	5, 300	3, 500, 000	1893	1	Gravel and rock	3, 900	2, 600, 000
1382	1	Limestone	6, 100	4, 284, 000	1915	7	Quartz gravel	3, 650	2, 650, 000
1409	4	Trap	3, 700	4, 340, 000	1921	11	Sandstone	3, 150	2, 360, 000
1432	1	Limestone	4, 800	3, 080, 000	1930	10	Mixed rock	4, 430	3, 120, 000
1444	3	Limestone and gravel	4, 100	2, 860, 000	1933	10	do	4, 020	2, 560, 000
1455	4	Limestone	4, 150	3, 980, 000	1948	7	Trap	4, 350	3, 250, 000
1467	1	Quartz gravel	5, 400	4, 720, 000	1966	1	Limestone	3, 500	3, 720, 000
1473	1	Trap	4, 500	2, 560, 000	1995	2	Mixed gravel	3, 450	2, 200, 000
1489	6	Quartz gravel	5, 300	3, 880, 000	2015	1	Limestone	3, 580	4, 000, 000
1501	5	do	4, 900	3, 560, 000	2041	2	Mixed rock	5, 450	5, 500, 000
1515	8	do	4, 300	2, 840, 000	2064	1	Limestone	4, 970	3, 980, 000
1524	2	Trap	4, 900	3, 280, 000	2085	1	do	6, 000	5, 700, 000
1536	3	do	3, 900	4, 320, 000	2112	½	Quartz gravel	3, 680	3, 180, 000
1564	½	do	5, 000	4, 360, 000	2135	2	do	5, 045	3, 600, 000
1576	5	Quartz gravel	4, 200	3, 200, 000	2160	1	do	3, 180	2, 400, 000
1591	7	do	3, 950	2, 120, 000	2185	1	do	3, 390	2, 840, 000
1601	2	Trap	5, 100	3, 560, 000					
1613	8	Quartz gravel	4, 900	5, 500, 000			Average	3, 960	3, 562, 000

---

[1] Possibly crushed gravel.

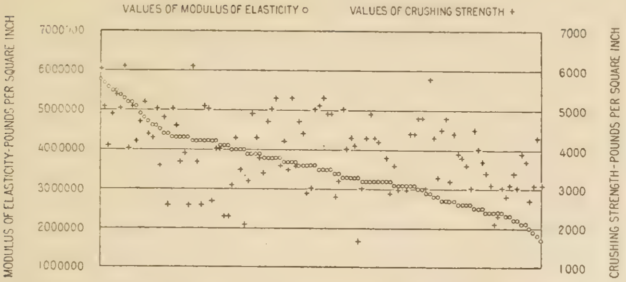

Fig. 5.—Modulus of Elasticity and Compressive Strength of Each Cylinder Arranged for Purpose of Comparison

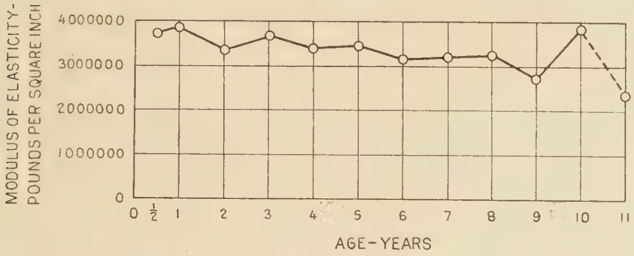

Fig. 6.—Average Values of Modulus of Elasticity of Concrete of Various Ages

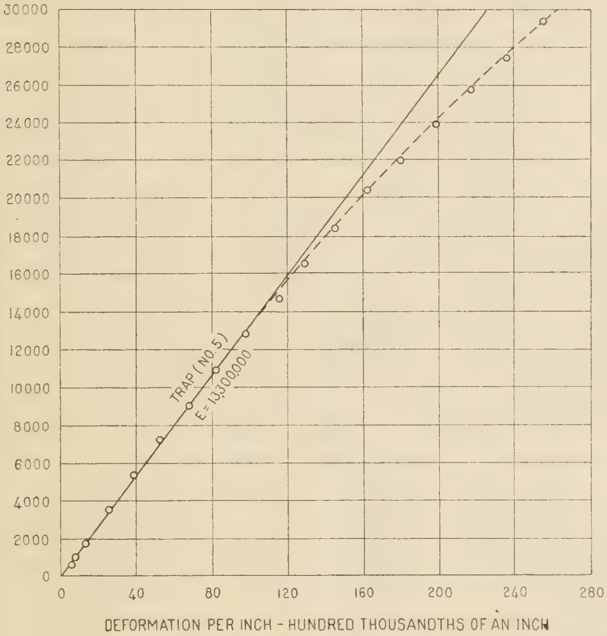

Fig. 7.—Elastic Curve for Trap Rock

The stress-strain curves for a number of the cores varying from one of the highest moduli to that of the lowest are shown in Figure 4. With one exception these curves show the same general characteristic, that is, the first portion of all of the curves is substantially a straight line, and then departing gradually from the straight line.

Special attention is directed to the curve shown for specimen 1870, where the elastic curve is convex with reference to the horizontal axis, all the others being concave. The aggregate in this core was quartz gravel, but this was the case also for specimens 1709, 1167, and 1790. This same characteristic of the stress-strain curve, convex towards the horizontal axis, was found for some sandstones and is particularly noticeable for concrete specimens when subjected to repeated loads.

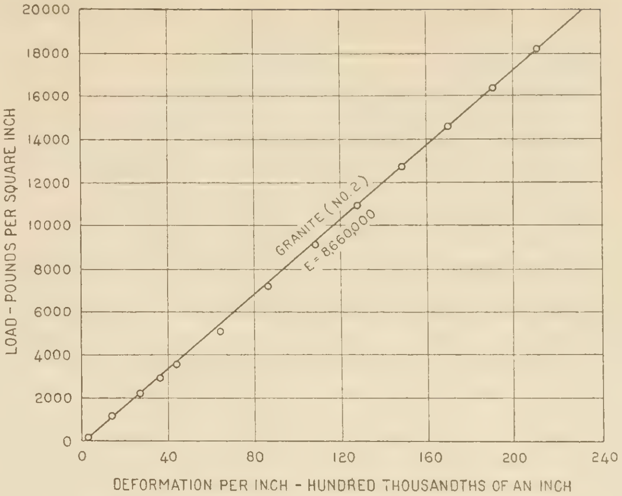

Fig. 8.—Elastic Curve for Granite Rock

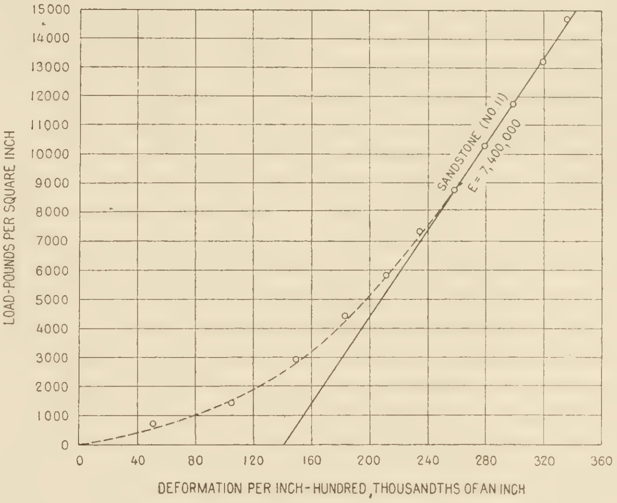

Fig. 9.—Elastic Curve for Sandstone

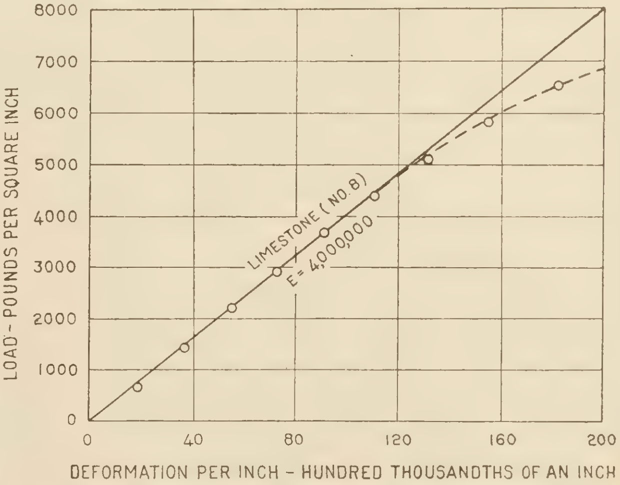

Fig. 10.—Elastic Curve for Limestone

A study of the curves shown in Figure 4 shows that the elastic limit fell as low as 400 pounds per square inch for specimen 541, and reached about 2,200 pounds per square inch for specimen 2015. A fair average seems to be in the neighborhood of 700 to 800 pounds per square inch. The results of these tests seem to indicate clearly that for this condition of loading the concrete in these specimens behaved as an elastic material.

A graphical presentation of the results of measurements of modulus of elasticity is shown in Figure 5, where the values of the modulus for each specimen are arranged in order from the highest to the lowest value, as shown by the circles. The value of the crushing strength for each specimen is shown by a cross. No marked relationship is found in these data between the crushing strength of the cores and the modulus of elasticity. For the 15 or 20 highest values for the modulus of elasticity, the corresponding values for crushing strengths are somewhat greater than the crushing strengths for the group of 10 specimens having the lowest modulus of elasticity. It is seen that the moduli fall for the most part between values of 2,400,000 and 4,400,000.

Figure 6 is a diagram showing values for the modulus of elasticity plotted according to the age of the specimens. It should be kept in mind that there was but one 11-year specimen but for the other years there were greater numbers. There is a slight trend towards a decrease in the modulus of elasticity with increase in age. However, it is felt that these data are not sufficient to justify any conclusion other than that the age of the concrete from one-half year on seems to have little effect on the value of the modulus of elasticity.

ELASTIC PROPERTIES OF ROCK INVESTIGATED

Results secured on concrete cylinders indicated the desirability of investigating the elastic properties of different kinds of rock. For this purpose the Bureau of Public Roads supplied three cylindrical specimens for each of four varieties of rock—limestone, trap, granite, and sandstone. These specimens were 2 inches in diameter and 4 inches long. The ends of the specimens were carefully ground to a flat surface. The gauge length over which the deformations were measured was 3 inches. The mirror type of extensometer was used.

As data on the elastic behavior of rocks are rare, the deformations for the various loadings of the different rock specimens in this series are given in Tables 2 and 3, while Figures 7 to 10 show the typical graphs of these data, the specimens selected for the typical graphs being the ones nearest to the average for the three specimens.

The marked difference between the stress-strain curves for sandstone and the other rocks is seen at once by reference to the corresponding graphs. All of the stress-strain curves other than that for sandstone are concave towards the horizontal axis, while that for sandstone is markedly convex with reference to the horizontal axis. This is the only stress-strain curve where the initial portion of the curve is not substantially a straight line, and it is also the only one where the initial portion of the curve is that of the least slope. The stress-strain curve for the sandstone becomes steeper as pressures increase. The slopes at loads of 10,000 pounds per square inch or approximately one-fourth of the ultimate strength of these specimens were obtained from the graphs and these values reported in the table as moduli of elasticity.

TABLE 2.—*Results of tests of rock cores 2 inches in diameter and 4 inches long*

Sample No.	Character of rock	Crushing strength	Modulus of elasticity	Elastic limit
		Lbs. per sq. in.	*Lbs. per sq. in.*	*Lbs. per sq. in.*
1	Granite	29, 900	8, 500, 000	(1)
2	do	32, 900	8, 660, 000	(1)
3	do	30, 500	8, 930, 000	(1)
4	Trap	35, 800	11, 400, 000	11, 000
5	do	34, 200	13, 300, 000	12, 000
6	do	34, 900	14, 200, 000	14, 000
7	Limestone	6, 620	3, 400, 000	4, 500
8	do	6, 990	4, 000, 000	4, 500
9	do	6, 620	4, 200, 000	4, 500
10	Sandstone	46, 000	[2] 7, 400, 000	------
11	do	39, 000	[2] 7, 400, 000	------
12	do	32, 200	[2] 6, 200, 000	------

(1) Straight line relationship throughout extent of observations.
[2] Value at 10,000 pounds per square inch (see text).

TABLE 3.—*Compressive deformations for rock cores*

Load, pounds per square inch	Granite			Load, pounds per square inch	Trap		
	Deformation in hundred-thousandths inch per inch				Deformation in hundred-thousandths inch per inch		
	No. 1	No. 2	No. 3		No. 4	No. 5	No. 6
0	0	0	0	0	0	0	0
73	0. 7	0. 5	------	735	------	6. 3	4
146	1. 2	1. 5	------	1,100	8	8	4
219	2. 3	2. 2	------	1,840	------	13. 2	9
292	3. 7	3. 3	------	2,200	16	------	------
365	4. 2	4. 3	------	3,300	27	------	------
547	7. 0	6. 5	------	3,680	------	26	20
730	9. 5	8. 8	------	4,410	36	------	------
912	11. 8	12. 0	------	5,520	47	39. 7	33
1,100	14. 3	13. 8	21	7,350	64	53. 8	47
1,460	19. 5	18. 2	------	9, 175	81	68. 5	60
2,200	28. 7	27. 0	33	11,000	99	83. 3	75
2,900	37. 3	35. 7	------	12,900	116	99	90
3,300	------	------	45	14,700	133	117	104
3,650	47. 8	44. 0	------	16,600	151	131	120
4,400	------	------	57	18,400	166	148	135
5,500	67. 8	65. 3	70	20,200	184	165	151
7,325	89. 2	86. 5	91	22,000	201	182	167
9,150	111	108	111	24,000	218	200	184
11,000	132	128	132	25,800	------	218	201
12,800	152	149	152	27,600	------	238	219
14,650	172	170	172	29,400	------	258	238
16,500	193	191	192				
18,300	213	------	214				
20,200	234	------	234				
23,900	------	------	275				
25,800	------	------	297				
Breaking load—pounds per square inch	29, 900	32, 900	30, 500	Breaking load—pounds per square inch	35, 800	34, 200	34, 900

Load, pounds per square inch	Limestone			Load, pounds per square inch	Sandstone		
	Deformation in hundred-thousandths inch per inch				Deformation in hundred-thousandths inch per inch		
	No. 7	No. 8	No. 9		No. 10	No. 11	No. 12
0	0	0	0	0	0	0	0
735	------	20	17	735	56	51	47
1,100	36	------	------	1,470	86	105	87
1,470	------	38	36	2,210	------	------	117
2,200	68	56	50	2, 940	128	150	142
2,940	------	73	67	3,670	------	------	165
3,300	99	------	------	4,420	159	183	184
3,670	------	92	86	5,150	------	------	202
4,400	130	111	105	5,880	185	211	218
5,150	------	132	123	6,620	------	------	234
5,500	166	------	------	7,350	207	236	249
5,880	------	154	148	8, 100	------	------	263
6,620	------	181	191	8,840	229	259	279
7,350	------	------	------	9,560	------	------	289
				10,300	249	280	300
				11,000	------	------	313
				11,750	268	300	324
				12,500	------	------	336
				13,240	287	320	347
				14,000	------	------	358
				14,700	305	338	------
				16,200	322	357	------
Breaking load—pounds per square inch	6, 620	6, 990	6, 620	Breaking load—pounds per square inch	46, 000	39, 000	32, 200

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.

DEPARTMENT BULLETINS

No. 105D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136D. Highway Bonds. 20c.
220D. Road Models.
257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370D. The Results of Physical Tests of Road-Building Rock. 15c.
386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387D. Public Road Mileage and Revenues in the Southern States, 1914.
388D. Public Road Mileage and Revenues in the New England States, 1914.
390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
463D. Earth, Sand-clay, and Gravel Roads.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537D. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
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*660D. Highway Cost Keeping. 10c.
*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
*724D. Drainage Methods and Foundations for County Roads. 20c.
*1077D. Portland Cement Concrete Roads. 15c.
1259D. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279D. Rural Highway Mileage, Income, and Expenditures, 1921 and 1922.

DEPARTMENT BULLETINS—Continued

No. 1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

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331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETIN

No. 55. Highway Bridge Surveys.

MISCELLANEOUS CIRCULARS

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93M. Direct Production Costs of Broken Stone.
*105M. Federal Legislation Providing for Federal Aid in Highway Construction and the Construction of National Forest Roads and Trails. 5c.

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*849Y. Roads. 5c.
914Y. Highways and Highway Transportation.
937Y. Miscellaneous Agricultural Statistics.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Connecticut.
Report of a Survey of Transportation on the State Highway System of Ohio.
Report of a Survey of Transportation on the State Highways of Vermont.
Report of a Survey of Transportation on the State Highways of New Hampshire.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL AID ROAD CONSTRUCTION

AS OF

SEPTEMBER 30,1928

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
				Initial	Stage[1]	Total			Initial	Stage[1]	Total		
		$	$				$	$				$	
Alabama	1,808.9	5,304,174.62	2,637,273.98	324.5	37.9	362.4	114,846.57	57,423.28		5.8	5.8	1,368,041.83	Alabama
Arizona	884.1	1,337,404.83	1,147,115.87	61.4	2.4	63.8	89,976.13	65,088.70		11.7	11.7	2,705,368.01	Arizona
Arkansas	1,693.0	4,834,561.96	2,152,800.63	169.5	6.3	175.8	10,511.44	5,255.72		.3	.3	1,745,940.42	Arkansas
California	1,500.9	7,422,743.56	3,446,173.34	161.9	8.7	170.6	692,192.90	294,745.49	24.9		24.9	2,491,946.69	California
Colorado	983.2	5,313,232.10	2,730,361.62	193.0	25.2	218.2	425,767.21	231,601.38	20.2		20.2	2,051,102.15	Colorado
Connecticut	207.6	2,869,898.44	607,411.53	34.2		34.2	179,362.16	39,975.00	2.7		2.7	566,752.61	Connecticut
Delaware	203.2	499,461.60	178,885.54	13.0	2.1	15.1	111,232.00	40,800.00	2.7		2.7	154,677.22	Delaware
Florida	403.5	3,310,473.40	1,418,070.87	94.5	5.5	100.0	777,845.42	269,730.00	18.0		18.0	1,212,651.02	Florida
Georgia	2,425.8	5,883,404.68	2,635,436.35	241.2	62.4	303.6	759,853.43	357,836.57	38.9	8.4	47.3	13,908.38	Georgia
Idaho	988.9	2,542,711.45	1,516,310.55	159.8	56.4	216.2	176,672.97	104,114.59	17.6		17.6	78,136.97	Idaho
Illinois	1,722.9	21,848,119.27	10,005,665.79	685.6		685.6	1,369,226.21	684,381.99	46.5		46.5	11,640.17	Illinois
Indiana	1,129.5	9,285,361.26	4,426,660.07	282.0	3.5	285.5	770,200.00	372,510.00	24.4		24.4	167,949.39	Indiana
Iowa	2,869.1	6,363,179.89	2,700,764.89	92.7	160.2	252.9	595,767.19	231,211.52	3.7	35.0	38.7	171,313.77	Iowa
Kansas	2,266.9	5,350,707.47	2,123,960.53	287.5	12.0	299.5	2,749,481.31	1,033,988.97	187.6	4.3	191.9	216,300.62	Kansas
Kentucky	1,181.4	5,297,880.28	2,521,712.10	225.2		225.2	1,118,397.65	536,198.82	71.5		71.5	223,556.30	Kentucky
Louisiana	1,276.1	4,884,408.20	2,434,445.45	198.3		198.3	103,055.30	25,000.00	.1		.1	185,163.73	Louisiana
Maine	442.2	2,086,381.56	714,744.58	49.2		49.2	876,028.62	335,765.43	27.5		27.5	896,807.45	Maine
Maryland	557.6	1,752,757.42	790,775.00	70.3	7.2	77.5						39,571.23	Maryland
Massachusetts	521.9	4,293,127.52	1,384,475.24	84.5		84.5	287,259.44	66,750.00	4.4		4.4	1,609,439.75	Massachusetts
Michigan	1,354.2	13,766,081.90	5,833,643.08	357.8		357.8	928,320.00	394,660.00	21.2	6.5	27.7	275,181.95	Michigan
Minnesota	3,844.9	6,234,006.97	2,074,316.22	276.8	91.4	368.2	358,602.89	118,000.00	5.8		5.8	192,755.21	Minnesota
Mississippi	1,603.7	4,722,654.74	2,125,568.92	184.4	31.6	216.0	27,945.22	13,972.61	3.3		3.3	565,056.46	Mississippi
Missouri	2,212.3	6,010,012.81	2,552,773.25	174.4	58.1	232.5	1,014,971.25	476,321.40	31.1	3.9	35.0	898,511.85	Missouri
Montana	1,458.2	3,603,613.91	2,209,512.83	284.3	13.4	297.7	791,857.26	416,558.56	81.7	4.1	85.8	4,090,763.15	Montana
Nebraska	3,339.9	4,820,684.06	2,390,276.17	461.8	101.3	563.1	373,768.35	186,305.80	14.8	41.9	56.7	1,828,629.86	Nebraska
Nevada	1,047.2	954,333.74	829,142.44	97.1	40.4	137.5	181,241.32	158,984.83	20.3	27.6	47.9	346,408.95	Nevada
New Hampshire	314.4	867,850.07	347,797.51	24.5		24.5	67,382.27	24,135.00	1.6		1.6	19,527.60	New Hampshire
New Jersey	433.5	4,833,648.27	854,202.35	58.9		58.9	868,654.88	124,005.00	8.3		8.3	86,765.94	New Jersey
New Mexico	1,765.1	3,111,037.07	2,026,907.93	207.3	.5	207.8	305,322.67	192,867.66	10.7		10.7	480,360.59	New Mexico
New York	1,899.1	32,109,200.00	7,354,627.50	481.2	8.6	489.8	6,446,900.00	1,309,305.00	87.4		87.4	3,451,909.55	New York
North Carolina	1,593.2	2,567,345.27	1,246,814.03	93.4	25.4	118.8	559,926.80	276,073.30	25.8	7.2	33.0	614,095.71	North Carolina
North Dakota	3,384.0	3,226,329.77	1,459,576.27	555,0	170.1	725.1	782,794.90	308,488.25	125.3	79.6	204.9	349,698.92	North Dakota
Ohio	1,839.8	13,772,270.18	5,031,678.85	309.0	6.0	315.0	3,519,880.00	1,095,346.04	60.0	12.7	72.7	1,554,811.86	Ohio
Oklahoma	1,641.9	4,014,297.77	1,860,360.11	180.5	14.3	194.8	991,180.05	408,499.79	50.7	12.4	63.1	193,632.76	Oklahoma
Oregon	1,121.5	1,166,341.49	683,052.57	28.8		28.8	208,344.40	106,849.33	14.1		14.1	1,231,794.83	Oregon
Pennsylvania	1,920.0	15,449,772.28	4,306,660.42	265.6		265.6	1,785,138.08	578,980.30	35.2		35.2	898,893.00	Pennsylvania
Rhode Island	150.5	871,885.43	234,446.25	14.7		14.7	112,609.50	43,974.55	1.6		1.6	583,494.49	Rhode Island
South Carolina	1,650.2	8,299,326.48	1,802,073.44	190.6	123.3	313.9						64,366.58	South Carolina
South Dakota	2,983.7	3,245,632.18	1,741,163.69	519.4	91.2	610.6	342,012.94	188,107.04	106.8	15.3	122.1	252,177.06	South Dakota
Tennessee	1,034.6	6,754,586.32	2,754,987.48	149.5	49.6	199.1	1,725,870.64	511,900.18	6.5	44.7	51.2	305,447.33	Tennessee
Texas	5,967.4	10,515,738.24	4,252,859.95	239.7	189.2	428.9	6,340,697.10	2,544,170.10	324.7	112.9	437.6	2,450,944.45	Texas
Utah	837.9	1,702,929.14	1,162,668.01	87.5	9.3	96.8	444,778.73	322,377.46	39.1	7.9	47.0	41,148.02	Utah
Vermont	219.0	1,175,591.63	384,026.29	23.6		23.6	335,700.10	71,725.00	7.4		7.4	42,653.83	Vermont
Virginia	1,272.9	4,472,888.08	1,430,846.87	115.4	21.6	137.0	981,946.61	178,222.56	33.6		33.6	11,604.44	Virginia
Washington	806.2	3,574,021.75	1,166,775.25	79.7	18.1	97.8	1,127,376.13	449,756.27	22.9	5.0	27.9	446,548.83	Washington
West Virginia	648.6	2,416,841.37	1,022,111.61	85.4	2.5	87.9	674,418.94	308,698.26	15.7	9.9	25.6	248,905.80	West Virginia
Wisconsin	2,118.0	7,651,724.51	2,726,962.29	233.6	32.6	266.2	430,778.33	213,640.00	14.8		14.8	1,359,870.96	Wisconsin
Wyoming	1,551.7	2,095,030.72	1,309,881.27	218.6	.6	219.2	87,529.10	19,260.00	9.4		9.4	7,971.18	Wyoming
Hawaii	39.4						175,931.99	57,501.20	1.8		1.8	1,066,498.59	Hawaii
TOTALS	73,119.7	274,485,665.66	108,748,746.78	9,426.8	1,488.9	10,915.7	43,199,556.40	15,851,062.95	1,672.3	457.1	2,129.4	39,870,696.46	TOTALS

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[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.